HDR

Memorandum

To: Frank Yanagimachi

From: Joyce Lem, Steve Aisaka

Date: July 16, 2002

Subject: **Use of the High Level West Seattle Bridge to Support the Monorail**

Submitted herein are results of our technical review of the ETC's proposed plans to use the West Seattle High Level Bridge to support a Monorail crossing of the Duwamish Waterway. The assessment was performed in accordance with the preliminary scope of work that was faxed to HDR on June 21, 2002. The scope of our work included reviewing the proposed structural design criteria, performing technical review of proposed design concepts, and reviewing estimates prepared by the ETC and its consultants.

EXECUTIVE SUMMARY

As directed, HDR has reviewed the ETC's proposed design concept(s), associated cost estimates, and design criteria developed for supporting the Monorail structure on the existing West Seattle High Level Bridge. HDR found no fatal flaws in the concept of supporting the Monorail structure on the West Seattle Bridge. The concept is technically feasible in general although there are areas of concern that should be considered.

Review Of Design Criteria

Further clarification of the Memo of Understanding (MOU) between the City and the ETC regarding seismic behavior of the existing bridge is warranted. It is unclear how the capacity/demand ratio should be applied to the structure when the original design was governed by displacement capacity.

Review Of Design Concepts

There are additional design calculations that should be performed to verify that the strengthening design concepts will be feasible on both the Main Span box girder bridge and the Harbor Island approach structures of the West Seattle Bridge. Constructability of the strengthening concepts should also be investigated further. Location of the transition between self-supporting and bridge-supported Monorail structures is variable and has not been set at the time of our review. We also recommend that the pile capacities of the existing foundations be evaluated.

Review Construction Cost Estimates

Our review of the construction cost estimate found no costs that were outside of industry norms. We feel that a contingency for uncertainty of construction costs and design concepts should be included in the construction cost estimates. For example, where the beginning and end of the bridge-supported Monorail structure is located will affect the costs of retrofitting the existing bridge and the costs of self-supporting Monorail structure. If a greater length of the Monorail structure is supported by the existing bridge, greater costs for retrofitting the bridge will be incurred. On the other hand, if a greater length of the Monorail structure is self-supporting, additional costs for new columns and foundations will be incurred. Bridge retrofit costs would be less and there may be additional costs for environmental mitigation and permitting for the self-supporting Monorail structure.

In summary, we recommend that the City remain closely involved in the development and evaluation of the concepts and design criteria as well as the design of the bridge-supported Monorail structures. We are under the impression that the ETC plans to construct the Monorail system under a design/build method of contracting. In a design/build atmosphere much of the control over the design decisions will be turned over the design/builder. While the design/build RFP documents can guide the design of the structure, the City may lose much of the traditional control of the design performed by the design/builder.

ETC'S STRUCTURAL CONCEPT FOR THE MONORAIL SUPPORTED BY THE EXISTING BRIDGE

For this study, it was assumed that the Monorail structure would be supported by the existing West Seattle Bridge (WSB) starting at Pier 15, the west end of the High Level Bridge (HLB), a twin single-cell concrete box girder bridge. The east end of the supported structure would be at the Harbor Island Approach (HIA) at either Pier 25 or 28, a five-span precast girder structure.

The proposed Monorail structure evaluated in this review consists of twin guideway beams above the roadway of the WSB. The beams are supported by a single column with crosshead, with the column supported at the roadway deck level of the WSB. Where support by the WSB ends, i.e., the transition to totally new substructure for the Monorail, a two-column straddle bent is proposed. Two straddle bent alternatives were considered by the ETC: either supported entirely on the WSB or with one column of the bent supported by the WSB and one with its own foundation.

The HLB is a cast-in-place segmental, post-tensioned, twin box girder with common deck slab. The deck slab is post-tensioned transversely. Span configuration is 375'-590'-375', with end spans supported by bearings on a pier common to the approach structures. The Monorail concept consists of 3 spans @ 125' along each end span and 4 @ 147.5' along the main 590' span. The Monorail column is located along the bridge centerline where there is median traffic barrier. A new post-tensioned diaphragm connecting the two box girders would be constructed under each of the mid-span Monorail columns to transmit the Monorail column reactions to the existing bridge box girders. Strengthening of the box girders for flexure and, perhaps, shear capacity would be required.

The HIA structures under consideration are the two 5-span structures to the east of the HLB. Spans lengths range from 154' to 164', with typical spans being 158.5' and 164.0'. The superstructure consists of precast, pre-tensioned girders with a composite concrete deck slab. The girder ends are cast in the pier cap beams so the structures behave as rigid frames. At the HIA structures the Monorail spans would match the existing bridge spans so the Monorail columns are seated on the existing cap beams. Strengthening of the cap beams would be required.

Foundation and column retrofit of the existing structure was assumed to be unnecessary in the proposed concepts.

BASIS FOR REVIEW

The information we received for review included:

- Seismic calculations by John Clark for the Harbor Island structures with many of the original Harbor Island structure plans
- Preliminary gravity loads for Monorail guideways on the existing WSB structures and miscellaneous calculations by Brian Garrett of BERGER/ABAM (B/A)
- Illustrations of structural concepts by B/A
- Some 1980 calcs for existing bridge elements
- Structural design criteria for the existing bridge, Seismic design criteria for the existing bridge and geotechnical basis/criteria
- Draft structural criteria for Monorail structures
- Two independent construction cost estimates
- Some of the Main Span (box girder) superstructure plans
- Draft Memoranda of Understanding (MOUs) proposed by the ETC regarding the existing structures' performance under the Monorail addition
- The paper "Seismic Design - West Seattle Bridge by Tom Mahoney and John Clark (1981)

Along with this information, we met with the City and ETC representatives at the beginning of our review (June 27, 2002) to review the MOUs and discuss aspects of the design concepts and construction cost estimates. We corresponded via email and phone with City staff and John Clark to clarify concepts and assumptions.

Our review is limited to review of the information listed above and exchanged via meetings and correspondence. We used the design concepts presented by the ETC as the basis for this review.

TECHNICAL REVIEW OF DESIGN CRITERIA

In general we did not find any items in the design criteria that could be termed a "fatal flaw". We offer the following comments on the applicability of the design criteria:

1. Draft #5 of the MOU (July 2, 2002) regarding seismic behavior of the existing bridge: for seismic analysis and determination of adequacy of the existing structures, we concur that displacement ductility demand and capacity of the existing structural system is the method to be applied to the taller structures (box girder structure and adjacent HIA approach structure). However, the definition of capacity needs to be further defined. The minimum C/D value of 1.25 needs to be clarified or revised.

   At this writing it is not clear to the reviewers what was defined as "displacement capacity" or "failure" for the 1980 design of the original bridge. Current design codes for new bridges define performance objectives such as "No repairable damage," "Slight or easily repaired, or moderate damage," and "Prevent collapse," etc. More severe damage is accepted under larger, less likely seismic events. The larger the seismic event, the further the bridge sways, and the greater the concrete cracking damage sustained. For example, a displacement capacity/demand against collapse of 1.25 is probably too low of a value whereas a minimum C/D or 1.25 against moderate cracking may be acceptable. A minimum displacement C/D ratio cannot be established without knowing what the existing bridge displacement C/D ratio is.

   For the WSB, we recommend that what be considered is the severity of concrete cracking in the columns as well as collapse prevention under seismic loading. In turn, severity of the cracking is related to reinforcing steel strain and concrete strain, primarily at the top and bottom of the bridge columns where plastic hinges are expected to form. We recommend that the displacement demand of the existing structural system be evaluated using current methods (e.g., linear elastic modal analysis with cracked column sections) and the displacement capacity determined for specified steel and concrete strain levels. Displacement capacity should be evaluated, assuming appropriate column plastic hinges. Current strain compatibility methods should be used and overall structure displacements determined at specified reinforcing steel and concrete strains. The structures should be re-evaluated under the addition of the Monorail structure and its mass. For a start, the 500 year ground motion could be used. W*e recommend that this ground motion loading should at least be compared to the design loading for an independent Monorail structure and the performance criteria for an independent structure. It is quite possible that the existing bridge structures will "pass" the design seismic loading for new Monorail structures although with a somewhat lower "C/D" ratio.*

   This recommended approach to evaluating the HLB is similar to the design approach for the original bridge, as described in the paper "Seismic Design - West Seattle Bridge by Tom Mahoney and John Clark (1981). This paper is presented as an attachment to the Draft MOU and forms the basis of many of our comments regarding the bridge substructure. Differences in today's approach from that in the 1981 paper may include slightly different strain compatibility relationships and whether cracked column section properties were used in the analysis of displacement demands. As presented in that paper, the existing columns provide the rotational and, hence, displacement ductility that determines the displacement capacity of the structural system. The cap beams (crossbeams) and foundation are designed to enforce plastic hinge development at the top and bottom column sections, respectively. Typically, shear reinforcing of the columns is also a function of the plastic hinge moments for the column. Column shear and crossbeam and foundations should be reviewed using current design methodology to determine that they meet the original criteria.

As part of this discussion, we note that moment reduction factors, i.e., “R Factors,” are irrelevant because moment C/D’s or D/C’s are irrelevant for the columns. The existing columns as they are reinforced need to be assessed for their displacement ductility capacity.

Except for the text in italics, John Clark and Joyce Lem essentially agree on a displacement capacity approach per a phone call on 7/11/02. There are recently developed criteria regarding levels of reinforcing steel strain and concrete strain for different performance levels. That said, because of the original design approach, it is both John and Joyce’s opinions that the HLB will figure okay under seismic loading. For the HIA structures, the location of the transition and how the transition should be determined depends on seismic demand on the existing structure and other issues (see HIA structures below). This should be discussed further. At this time, a draft C/D ratio for lateral deflection might be considered, subject to refinement as the numbers are run. We feel that the City should require submittal and evaluation of any analysis of the existing bridge.

2. Applicable Codes: The codes listed should also include current interims. Add “and current interims” each reference of the AASHTO design specification.
3. Section 5.7 Deformations: Vertical deflection of the bridge spans should also not be greater than 1/1000 of the span length under Monorail LL + HS20.

TECHNICAL REVIEW OF DESIGN CONCEPTS

The review of the design concepts focused on two basic questions:

- Is there an adequate load path for non-seismic loading, i.e., can the existing bridge support the weight of the proposed Monorail structure and its “crush loaded” trains?
- How will the existing bridge behave under seismic loading?

In general, we found the concept of supporting the Monorail structures on the WSB to be technically feasible although we have concern over the following major issues:

- Box girder strengthening design concept
- Design concept, including constructability, for the strengthening of the cap beams at the Harbor Island Approach structures
- Location of the transition between self-supporting and bridge-supported Monorail structure, especially at the Harbor Island Approach structures
- Validation of the pile capacities at all foundations

- Monorail support at Piers 15 and 18, the expansion piers of the Main Span box girder, and the details of the Monorail column support at the existing diaphragms.

We received two sets of concepts and associated construction cost estimates although the general scheme for Monorail support by the WSB was as described in above. Some of our comments concern feasibility of one or the other.

Uncertainty about these items should be reflected in cost estimates. Further investigation may be warranted to insure the feasibility of the project.

The text below addresses the major issues listed above. The HLB is discussed first, followed by the HIA structures. Non-seismic load issues and seismic load considerations for each are presented.

High Level Bridge (HLB)

1. The flexural check of the box girders was performed using ultimate loads. The design of the segmental post-tensioned box structure was done in working stress. To be consistent with the original design, the service load stresses should be checked when retrofitting the bridge. Service load capacity will likely control the design.

2. The analysis should consider the secondary post-tensioning effects that will be imparted to the existing bridge when the Monorail beams are stressed.

3. Two diaphragm retrofit concepts were presented. The diaphragm concept presented by Brian Garrett at the mid span Monorail column supports appears to be inadequate in transferring longitudinal moments from the Monorail columns to the HLB. The concept proposed by Tom Mahoney -- a box beam connecting the two box girders -- is more likely, in our opinion. The proposed box beam would have a solid diaphragm directly below the Monorail column.

4. We were unable to verify the 50,700 ft-kips (BEG's calculations) of additional capacity that will be required to retrofit the bridge for the Monorail. From John Clark's calculations, at midspan of the main span Mu = 319,430 ft-kips, ϕMn = 229,334 ft-kips. A difference of 90,000 ft-kips. Additionally a live load shear of 126 kips should be used in the middle span shear check.

5. The two web strengthening concepts as proposed by B/A and Mahoney were discussed at the June 27 meeting. It is thought that that B/A concept is not feasible due to congestion in the web and the length and orientation of core drilling required.

6. A substantial dead load is being added to the existing structure. The additional Monorail structure, diaphragms and post tensioning may have an effect on the rideability of the existing roadway surface due to additional deflection and long term creep. The short term and long term deflections due to loads from the Monorail structure should be considered.

7. Details for the transfer of Monorail column loads at or near the expansion columns of the HLB were not considered in very much detail. On the conceptual plans a column has been located directly over the expansion joint between the HLB and the HIA. Loads from the Monorail column will presumably

be transferred to the existing structure through a diaphragm similar to that shown on Berger/ABAM's plans or as described by Tom Mahoney (see Item 3 above). On Drawing S-41 of the original plans, the longitudinal section at Pier 18 is shown schematically. Both diaphragms for the HLB and HIA superstructures are supported by the common pier cap and, hence, are “narrower” than a typical cap beam at an interior pier. Retrofitting of this cap beam is included in the Mahoney estimate. The additional dead and live load moments due to the eccentricity of the bearings should be considered in the checking of the expansion columns and foundations.

8. Foundations: A spot check of vertical loads on the piles was performed for Piers 16 and 17 – the following shows the increase in DL and LL is relatively small compared to existing bridge loads. Furthermore, the Seismic Design paper by Mahoney and Clark stated that seismic loads governed the foundation design. We did not have detailed plans for Piers 16 and 17 substructure or foundations, but found calculations for the existing HLB and note the following:

Pier 16 or 17 Foundation Loads For Entire Bridge Width (i.e., Loads for both columns) [kips]		
	Existing HLB	Monorail Addition (2 tracks)
DL	39,000	2,420
LL	2,700	330
Total	41,700	2,750

- 2750 / 41700 = 6.6% in vertical load

During final design, review plans of existing structure, pile data and report on long-term vertical capacity of the piles that was done by Shannon & Wilson, Inc. (HDR does not have this document) to confirm pile capacity and total non-seismic loads.

9. Seismic design issues: Per the paper Seismic Design paper by Mahoney and Clark (1981), the original HLB substructure design was governed by seismic loading, not the gravity loading as discussed in the preceding Item 6. The design approach outlined in that paper is similar to what is used today: Seismicity and soils were evaluated and anticipated ground motions and response spectra developed. Ground motions were applied to the structure using response spectrum analysis and time history analyses. Column flexural design was based on loads derived from these analyses. Plastic hinge moments at the top and bottom of the columns were used to design the foundations and cap beams and shear reinforcing for the columns. This design approach ensures ductile behavior of the structure, that plastic hinging can be developed in the columns at controlled locations. Based on the Seismic Design paper, the seismic design of the HLB and detailing of the columns is similar to what is done in today’s designs. This should be reviewed in detail to ensure that the design is per the Seismic Design paper. Although a summary of the displacements was included in the materials we received, we didn’t find a discussion of displacement capacity. See discussion on Seismic Design Criteria above.

In general, the HLB structure is a tall and flexible structure. Under the design acceleration of 0.32g (500 year return period), the approximately 120-ft tall columns deflect on the order of 1-ft, according to the original bridge calculations we received. For preliminary design, the addition of the Monorail structure can be treated as additional mass at the deck level so in general, somewhat larger translations at the deck might be expected under the same seismic loading assumed for the original HLB. The net increase in mass appears to be on the order of 9% (approximately 6,300 kip Monorail added to 71,800 kip existing superstructure). Although somewhat larger displacements might be expected under the larger mass, it is anticipated that these are well within the displacement capacity of the existing structure. We note that the response spectra used for the original bridge design shows a constant displacement spectra of 0.81 ft for modes with periods greater than approximately 2.5 seconds.

A displacement-based approach should be used in any final design to evaluate the existing structure. Current methods developed in the last few years should be used to evaluate acceptable strains in the concrete and reinforcing steel of the columns. That said, it is our opinion that the HLB columns and foundations will not require retrofitting under seismic loading with the addition of the proposed Monorail structure because they are designed and detailed based on plastic moment capacity of the columns. And correspondingly, the construction cost estimates do not include any retrofit of the columns or foundations

Harbor Island Approach Structures:

10. Brian Garrett (B/A) and Tom Mahoney/John Clark differ in opinion as to where the structure transitions from being supported by the existing Harbor Island approach structure and being self-supporting. Tom/John indicate Pier 25 should be the easternmost existing pier and Brian used Pier 28. Where the transition takes place will need to be finalized. Additional construction and environmental costs will be incurred as the transition is located westward, i.e., more new structure would be constructed, probably in the waterway.

11. Crossbeams:

    - No check of the shear capacity of the existing crossbeam was performed and flexure moment calculations were not furnished although, according to the design criteria for the original HIA, the crossbeams were designed for the plastic moment capacity of the columns. Both shear and flexure should be evaluated for the added vertical load due to the Monorail addition.

    - Constructability of the external post-tensioning should be verified. Critical issues include feasibility of access holes through the exterior girders, their location and missing the prestressing steel and mild steel reinforcing for shear in the girders.

    - Aesthetics of the retrofit may also become an issue because aesthetics of the piers was important during the original design.

12. Foundations: A spot check of vertical loads on the piles was performed for Pier 20, a typical interior bent at the approach structure adjacent to the HLB. During final design, there should be a review of the as-built plans, pile data and report on long-term vertical capacity of the piles that was done by Shannon & Wilson, Inc. (HDR did not review this report; it is thought to be in the City files) to confirm pile capacity and total non-seismic loads. Unlike the HLB, the increase in vertical loads at each of the foundations is a larger percentage of the capacity noted on the existing bridge plans. Validation of the foundation capacity/demand ratio under non-seismic loading should be provided to the City during final design. John Clark was optimistic that the allowable vertical loads on the piles may be larger than the 200 Tons noted in the design criteria and on the plans. Our rough calculations show the following for Pier 20:

Pier 20 Foundation Loads For One Column [kips]		
	Existing HIA	Monorail Addition (2 tracks)
DL	4,250	383
LL	210	144
Total	4,460	527

- 527 / 4440 = 12%
- Total Vertical Load on Piles = 4460 + 527 = 4987 kip.
- Existing 16 – 24" prestressed concrete piles @ 200 Ton each = 6,400 kips allowable vertical load (service level)

13. Seismic design issues: John Clark performed a longitudinal and transverse seismic analysis on the 5-span structure incorporating Piers 23 through 28, which we discussed with him. Design approach for the original HIA structure was similar to the HLB design approach. For this 5-span structure, the easternmost columns at Piers 27 and 28 are significantly shorter than the 4 westerly pier columns. The likely result under seismic loading is that Piers 27 and 28 will see disproportionately higher shears and moments, which is the rationale for Tom/John transitioning off of the HIA structure "sooner" than the B/A concept. Again, it is our opinion that the final design for the structures should evaluate displacement capacity versus displacement demand of the existing HIA columns using current criteria for acceptable strains in concrete and reinforcing.

REVIEW CONSTRUCTION COST ESTIMATES

The conceptual drawings and costs estimates prepared by Tom Mahoney and Brian Garrett were reviewed by Mike Loo of HDR's Construction Control Corporation (CCC). The CCC branch of HDR is routinely involved with construction estimating and Design/Build projects. The review of the construction cost estimate found no costs that were outside of industry norms. However, we recommend that a larger contingency factor be used in the estimate due to the lack of detailed engineering (as mentioned earlier in this memo) that has been performed at this point of project.

For example, where the beginning and end of the bridge-supported Monorail structure is located will affect the costs of retrofitting the existing bridge and the costs of self-supporting Monorail structure. If a greater length of the Monorail structure is supported by the existing bridge, greater costs for retrofitting the bridge will be incurred. On the other hand, if a greater length of the Monorail structure is self-supporting, additional costs for new columns and foundations will be incurred. Bridge retrofit costs would be less and there may be additional costs for environmental mitigation and permitting for the self-supporting Monorail structure.

HDR maintained the same assumptions that the project will ultimately be built through a traditional Design-Bid-Build method. Cost estimates evaluated herein are for modifications to the existing structures, i.e., they do not include Monorail columns or guideway construction. Cost estimates are for construction only and do not include design or construction management costs. Cost estimates are in Year 2003 dollars.

The two construction estimates furnished to us were developed independent of each other. In summary, the Mahoney estimate for existing bridge modifications was $17.44M and the B/A estimate for existing bridge modifications was $11.25M (including a 25% contingency). It is our opinion that the Mahoney estimate is closer to the actual costs based on our technical review . This estimate seemed to be more detailed in the various bid items necessary to complete the task, providing a higher level of confidence.

The B/A estimate used Means 2002 Heavy Construction data and there did not appear to be any escalation factors for increases in costs to 2003. The Mahoney estimate used early 2003 dollars as a basis and included escalation to mid-2003.

At this conceptual design level, and since this is a retrofit of an existing structure, the probability of a contractor encountering unforeseen conditions is high so we believe that a contingency should be used in the construction cost estimate, say 40%. Furthermore, the location of the transition between bridge-supported and self-supporting Monorail structure is not determined yet. Using the Mahoney estimate as a basis, the order of magnitude that should be considered for construction:

Total for Items 1 through 13:	$17.44M.
Contingency: increase to 40%	6.98M
Total:	$24.42 M
	Say $25M in mid-2003 dollars

We note that there is no retrofit of foundations included in either estimate. There is no allowance for the costs of environmental work that may be necessary for permitting the construction of either new foundations at the straddle bent proposed in the Mahoney estimate nor for retrofit of foundations.

The owner should also consider a “time and material” contingency bid item to quickly resolve issues arising in the field.

It is our opinion that costs of City review of the analysis of the WSB should be included in the Monorail costs.

It is recognized that neither HDR nor the City of Seattle has control over the cost of labor, materials, or the Contractor's methods of determining bid prices or market conditions. HDR cannot and does not warrant, represent, make any commitments, or assume any duty to assure, that bids or negotiated prices will not vary from any estimate of Construction Costs or evaluation prepared or agreed to by HDR.

Some comments on each of the estimates:

Mahoney Estimate:

Items 5,6-Main Span at Piers:
The descriptions for these items are general, so it was hard to determine what was included.

Items 11 thru 13:
These are lump sum items, which would typically be included in the unit cost for the respective items of work or in the bidders overhead costs. Item 13 is said to include labor escalation to mid-2003, material and equipment escalation, and incidental overtime. This is not contingency for uncertainty in costs or design concepts. An increase in the contingency is discussed above.

B/A Estimate:

This estimate included the cost to remove the existing median barrier and replace it with single faced half barrier. Can the existing double face median barrier be utilized except at the Monorail pier locations?